VOLUME 78 SEPARATE No. 155

PROCEEDINGS

Rochester Public Library
OCT 27 1952

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

115 South Avenue
ROCHESTER 4, N. Y.

OCTOBER, 1952

FIELD STUDY OF A SHEET-PILE BULKHEAD

By C. Martin Duke, A. M. ASCE

SOIL MECHANICS AND FOUNDATIONS DIVISION

Copyright 1952 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N.Y.

PRICE $0.50 PER COPY

r620.C
A512p.

GUIDEPOST FOR TECHNICAL READERS

"Proceedings-Separates" of value or significance to readers in various fields are here listed, for convenience, in terms of the Society's Technical Divisions. Where there seems to be an overlapping of interest between Divisions, the same Separate number may appear under more than one item.

Technical Division	*Proceedings-Separate Number*
Air Transport	42, 43, 48, 52, 60, 93, 94, 95, 100, 103, 104, 108, 121, 130, 148 (Discussion: D-23, D-43, D-75)
City Planning	58, 60, 62, 64, 93, 94, 99, 101, 104, 105, 115, 131, 138, 148, 151, 152, 154 (Discussion: D-16, D-23, D-43, D-60, D-62, D-65, D-86)
Construction	43, 50, 55, 71, 92, 94, 103, 108, 109, 113, 117, 121, 126, 130, 132, 133, 136, 137, 145, 147, 148, 149, 150, 152, 153, 154, 155 (Discussion: D-3, D-8, D-17, D-23, D-36, D-40, D-71, D-75, D-92, D-109, D-113)
Engineering Mechanics	122, 124, 125, 126, 127, 128, 129, 134, 135, 136, 139, 141, 142, 143, 144, 145 (Discussion: D-24, D-33, D-34, D-49, D-54, D-61, D-96, D-100)
Highway	43, 44, 48, 58, 70, 100, 105, 108, 113, 120, 121, 130, 137, 138, 144, 147, 148, 150, 152, 155 (Discussion: D-XXVIII, D-23, D-60, D-75, D-109, D-113)
Hydraulics	107, 110, 111, 112, 113, 116, 120, 123, 130, 134, 135, 139, 141, 143, 146, 153, 154 (Discussion: D-70, D-71, D-76, D-78, D-79, D-86, D-92, D-96, D-113)
Irrigation and Drainage	97, 98, 99, 102, 106, 109, 110, 111, 112, 114, 117, 118, 120, 129, 130, 133, 134, 135, 138, 139, 140, 141, 142, 143, 146, 148, 153, 154, 156 (Discussion: D-86, D-87, D-92, D-96, D-109)
Power	120, 129, 130, 133, 134, 135, 139, 141, 142, 143, 146, 148, 153, 154 (Discussion: D-38, D-40, D-44, D-70, D-71, D-76, D-78, D-79, D-86, D-92, D-96, D-109)
Sanitary Engineering	55, 56, 87, 91, 96, 106, 111, 118, 130, 133, 134, 135, 139, 141, 149, 153 (Discussion: D-29, D-37, D-56, D-60, D-70, D-76, D-79, D-80, D-84, D-86, D-87, D-92, D-96)
Soil Mechanics and Foundations	43, 44, 48, 94, 102, 103, 106, 108, 109, 115, 130, 152, 155 (Discussion: D-43, D-44, D-56, D-75, D-86, D-109)
Structural	117, 119, 121, 122, 123, 124, 125, 126, 127, 128, 129, 132, 133, 136, 137, 142, 144, 145, 146, 147, 150, 155 (Discussion: D-51, D-53, D-54, D-59, D-61, D-66, D-72, D-100, D-109)
Surveying and Mapping	50, 52, 55, 60, 63, 65, 68, 121, 138, 151, 152 (Discussion: D-60, D-65)
Waterways	41, 44, 45, 50, 56, 57, 70, 71, 96, 107, 112, 113, 115, 120, 123, 130, 135, 148, 154 (Discussion: D-8, D-9, D-19, D-27, D-28, D-56, D-70, D-71, D-78, D-79, D-80, D-113)

A constant effort is made to supply technical material to Society members, over the entire range of possible interest. Insofar as your specialty may be covered inadequately in the foregoing list, this fact is a gage of the need for your help toward improvement. Those who are planning papers for submission to "Proceedings-Separates" will expedite Division and Committee action measurably by first studying the ASCE "Guide for Development of Proceedings-Separates" as to style, content, and format. For a copy of this Manual, address the Manager, Technical Publications, ASCE, 33 W. 39th Street, New York 18, N. Y.

The Society is not responsible for any statement made or opinion expressed in its publications

Published at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York 18, N. Y. Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

Proceedings—Vol. 78 October, 1952 Separate No. 155

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

FIELD STUDY OF A SHEET-PILE BULKHEAD

By C. Martin Duke,[1] A. M. ASCE

Synopsis

Methods and results of a field study of the structural performance of a flexible anchored bulkhead are presented in this paper. The bulkhead retains a 55-ft height of fine sand hydraulic fill in Long Beach Harbor, California. A coarse granular dike extends for half the height on both inside and outside. Instrumentation was provided for the measurement of soil pressure, pore pressure, tie-rod tension, and deflection during and after the filling operation.

During filling, the lateral soil pressure distribution was approximately in proportion to the weight of the overlying fill, with a lateral pressure coefficient of approximately 0.7. After the completion of filling, the lateral pressures above the wale increased sharply and those below decreased. This change is attributed to the settlement of fill and the accompanying partial support of fill on the tie rods and wale.

The inner dike was effective in reducing outward soil pressures on the bulkhead. The outer dike provided some passive resistance to deflection and served to confine the bulkhead foundation.

Introduction

The anchored flexible bulkhead presents an example of the group of problems in foundations and earth structures that are not amenable to accurate analysis because of insufficient understanding of the performance of the prototype system. Performance of such bulkheads is poorly understood for the following reasons:

1. The bulkhead, the backfill, and the supports at the anchor and the toe all deform in accordance with as yet unformulated functions of soil and bulkhead load-deformation characteristics;

Note.—Written comments are invited for publication; the last discussion should be submitted by April 1, 1953.

[1] Associate Prof. of Eng., Univ. of California, Los Angeles, Calif.

2. The factors determining relations between lateral and vertical soil pressures even under idealized conditions are still incompletely understood; and

3. The influences on lateral pressures of such prototype phenomena as tide fluctuation, pore pressure, settlement of fill, and construction irregularities are not quantitatively known.

Efforts toward a determination of the fundamental parameters of bulkhead performance have led (as with many other soil problems) to the conclusion that further advance must lean heavily on experimental scientific studies of actual engineering structures. An opportunity for an investigation of this type arose when the Port of Long Beach, Calif., as part of its development program, planned the construction of two large hydraulically-filled piers bordered by anchored steel sheet-pile quay walls. This project led to the cooperative program of investigations which is the subject of this paper.

The structural performance of the east bulkhead of Pier C, Long Beach Harbor (Fig. 1) during and after hydraulic filling of the pier, was investigated

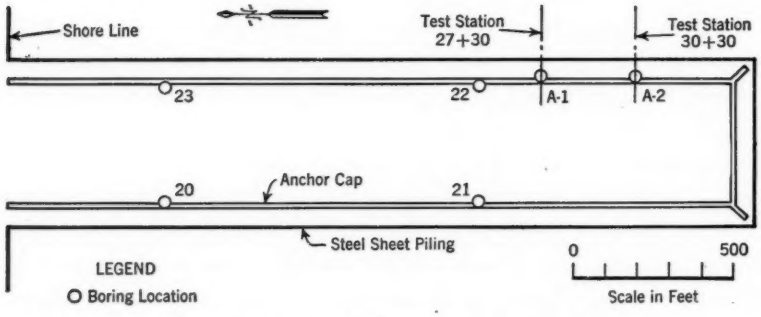


FIG. 1.—PIER C, LONG BEACH HARBOR, CALIFORNIA

by instruments installed at two test stations. The pier was filled to El. +17 and has an outboard bottom elevation of −38 at the test locations. A coarse granular dike was placed on both sides of the buklhead to El. −10 prior to filling. The fill material consisted predominately of a fine sand containing varying amounts of silt.

Provision was made at the principal test station for the measurement of total deflection of bulkhead and anchor, relative deflection at various elevations on the bulkhead, bending strain in the bulkhead, tension in tie rods, soil pressures at various elevations, and water pressures at significant locations in the fill. These measurements, except for soil and water pressure, were duplicated at another test station. Observations were made over a period of eight months, from February to October, 1949, with principal activity during the filling operation from February to June.

The investigation probed the distribution and magnitude of soil pressure against the bulkhead and its variation with height and age of fill. Also studied were the effectiveness of the dike in carrying a part of the lateral pressure and the pore pressure phenomena in the fill and dike. Special emphasis was

given to those aspects of the study that would yield information not obtainable quantitatively from laboratory, model, or theoretical analyses.

Structure and Materials

The Pier C quay wall (Fig. 2) consists of a bulkhead of 78-ft steel sheet piles (MZ-38) anchored by 3-in. tie rods at 6-ft intervals to a concrete-capped batter-pile anchor set 62 ft back of the bulkhead center line. A continuous wale consisting of two 15-in. channels is bolted to the inside of the wall. Tie rods are set on a slope of 1 on 27. The timber anchor piles are 3 ft, center to center, in two rows battered each way from the anchor center line. Tide range in Long Beach Harbor is about 8 ft between annual extremes; there are two high tides and two low tides each day.

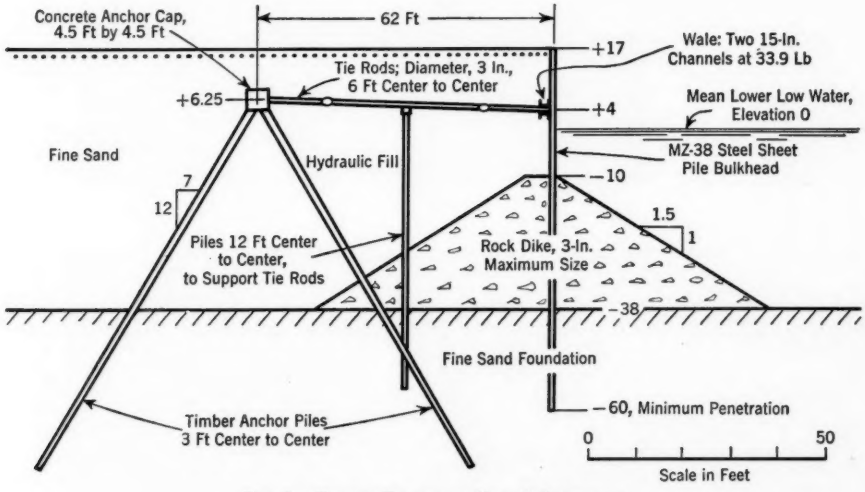


Fig. 2.—Design Features, Pier C Bulkhead

After the completion of pile driving, installing wale and tie rods, and concreting the anchor cap, the rock dike was placed by clamshell buckets on both sides of the wall, sloping at its natural angle of repose. The rock used in the dike was a partly decomposed diabase obtained from a quarry west of near-by San Pedro. The larger particles are angular and fairly hard (Fig. 3). Mechanical analyses showed 90% passing the 3-in. sieve and 15% passing the $\frac{1}{4}$-in. sieve. The angle of repose measured from the dike profiles was 32°.

Hydraulic filling from dredges working in the vicinity of the Los Angeles River delta was begun in mid-December, 1948, at the north or shore end of the pier and progressed seaward. Overflow was at the seaward end, first through the unclosed southeast corner of the pier and later through a sea-level spillway in the center of the south bulkhead. This procedure tended to produce a characteristic fill profile, grading from silty clay at the bottom to coarse sand at the top. In mid-February, 1949, deposition began at Test Station 27 + 30, and by mid-June, 1949, filling had been substantially completed at both test stations. Fig. 4 is a typical view of the fill.

FIG. 3.—DIKE MATERIAL AS DEPOSITED ON WALE AND DETAIL OF TYPICAL PORE PRESSURE POINT

FIG. 4.—FILL MATERIAL SHOWING CRACK OVER ANCHOR CAP DUE TO FILL SETTLEMENT

Samples were studied from sixty-nine borings in the harbor area. Four of the borings, numbers 20, 21, 22, 23, Figs. 1 and 5, were made in Pier C prior to filling, and extended to about El. −100. "Undisturbed" samples, 3 in. in diameter, were taken at 3-ft intervals in twenty three of the borings, including those in Pier C, and classification samples were taken in the remainder. Laboratory density, mechanical analysis, "quick" triaxial compression, permeability, and consolidation tests were made on a large number of samples representing all soil types encountered. Approximate tests were made to estimate maximum and minimum void ratios, and Atterberg limits were established for the few plastic soil types encountered. After completion of

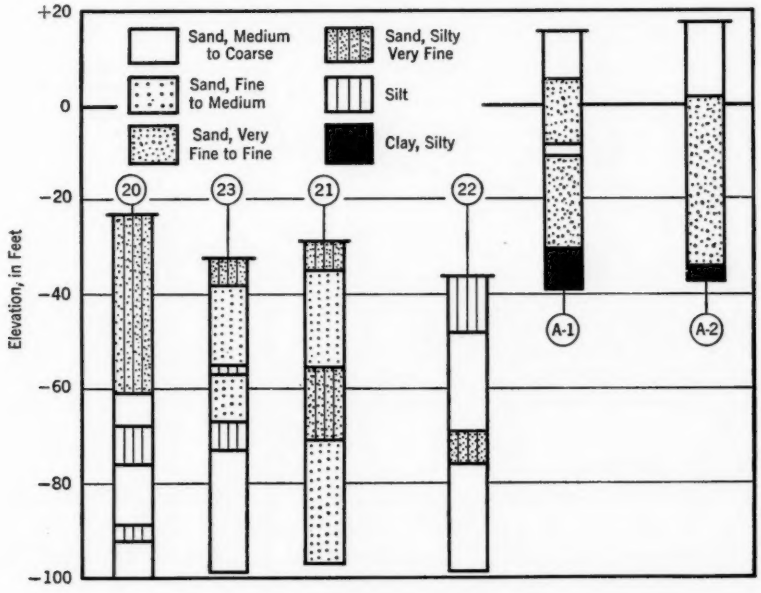


FIG. 5.—LOGS OF BORINGS

filling, borings A-1 and A-2, Figs. 1 and 5, were made through the fill opposite the test stations.

Soil property and stratification data as they pertain to Pier C at the test stations are summarized in Table 1, where data on dike properties also are included. The compressive index (decrease in void ratio per 100% increase in pressure on a preconsolidated specimen) for strata (b) and (d) were obtained from tests on similar soils in other parts of the harbor. Grain size limits are those of the United States Bureau of Chemistry and Soils.[2] For the fine sands of the fill, the permeability coefficient ranged between 10 and 35 $\times$ 10^{-4} cm per sec, and the percentage of the material passing sieve No. 200 ranged

[2] "Soil Mechanics," by D. P. Krynine, McGraw-Hill Book Co., Inc., New York, N Y., 1947, p 17.

between 5 and 15, approximately. Lenses of silt and of coarse and medium sands and occasional lumps of organic clay were present.

Instrumentation

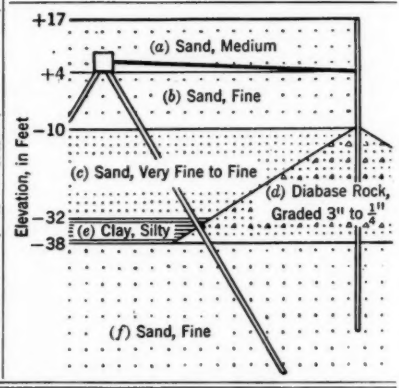
Scheme of Instrumentation.—Much study was given to the problem of how best to observe the behavior of the soil and bulkhead. General features of instrumentation are shown in Fig. 6. Measurements consisted of the following:

TABLE 1.—Summary of Soil Properties at Test Stations

Stratum (see Col. 8)	Specific gravity	Unit weight[a] (lb per cu ft)	Coefficient of permeability (cm per sec)	Angle of internal friction	Void Ratio	
					Representative	Range
(1)	(2)	(3)	(4)	(5)	(6)	(7)
(a)	2.67	105	100×10^{-4}	38°	0.8	0.6 to 1.1
(b)[b]	2.67	55	25×10^{-4}	37°	0.8	0.7 to 1.2
(c)	2.68	55	20×10^{-4}	34°	0.95	0.7 to 1.3
(d)[c]	2.42	60	....	(32°)	....	....
(e)[d]	2.72	48	....	13°	....	....
(f)	2.67	55	25×10^{-4}	37°	0.8	0.7 to 1.2

[a] All samples submerged except for stratum (a) which was semisaturated. [b] In stratum (b) the compressive index was 0.05. [c] The value 32° given in Col. 5 for stratum (d) is the angle of repose. [d] The silty clay in stratum (e) has a cohesion of 900 lb per sq ft and a compressive index value of 0.15. The liquid limit (LL) and the plastic limit (PL) were 48 and 31, respectively.

Description of Strata (See Col. 1) (8)



At Both Test Stations.—

1. The deflection of points on the upper half of the bulkhead was measured by reference to a plumbed vertical beam.
2. The tension in the tie rods was measured by pairs of unbonded electric strainmeters attached to three tie rods at each test station.
3. Fill settlement was determined by indirect methods.

At Station 27 + 30 Only.—

4. Pressure of the soil against the bulkhead was obtained with nine electric stressmeters. Four of these instruments were installed in the fill above the dike. Five were installed in the dike—three inside, and two outside, the bulkhead.
5. Water pressure on stressmeters and on the inside of the bulkhead, and pore pressure in the fill, were measured with six standpipe piezometers.

In addition, at both test stations, measurements were undertaken of the bulkhead bending moment by a mechanical strain gage and of the total deflection of bulkhead and anchor by transit. The precision and consistency of these measurements was so poor that they have been disregarded. Water and fill elevations were referred directly to points on the bulkhead in order to

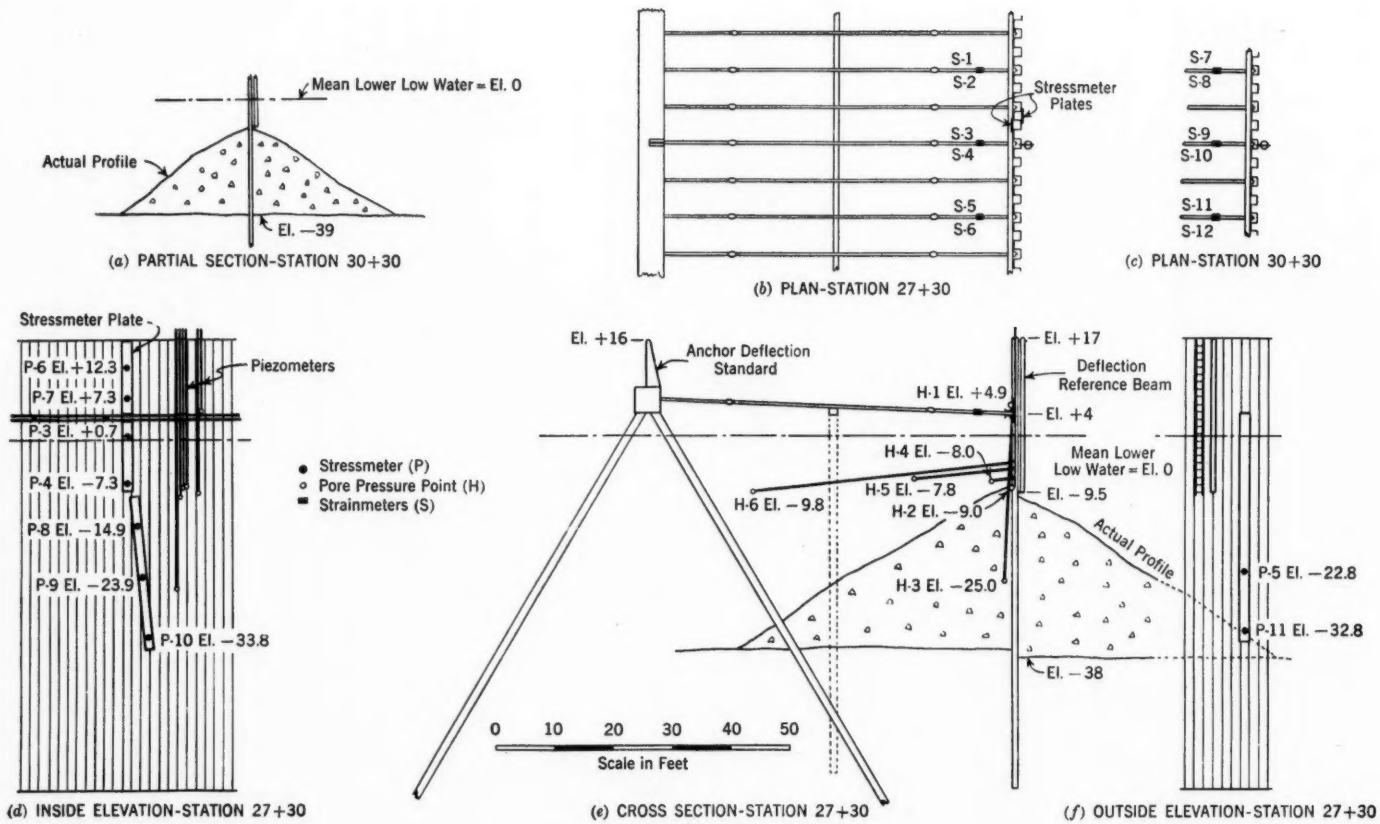


FIG. 6.—INSTRUMENTATION LAYOUT

eliminate errors due to a simultaneous regional subsidence. Nominal datum is "mean lower low water" (MLLW).

Observations were made three or four days apart during the placing of the fill, which occupied a period of about three months. On selected days, observations were made at 2-hr intervals during a maximum tidal range. Readings at decreasing frequencies were made up to October 5, 1949, when the test were discontinued because insufficient instruments remained active after this date to justify further observations.

Tie-rod Tension.—Six tie rods, three at Station 27 + 30 and three at Station 30 + 30, were instrumented with two strainmeters,[3,4] each, for the measurement of tie-rod tension. The strain-sensitive element of the strainmeter consists of two coils of wire, one of which stretches while the other shortens when the ends of the meter are displaced relative to one another. A portable Wheatstone bridge set is used to observe the ratio of the resistances

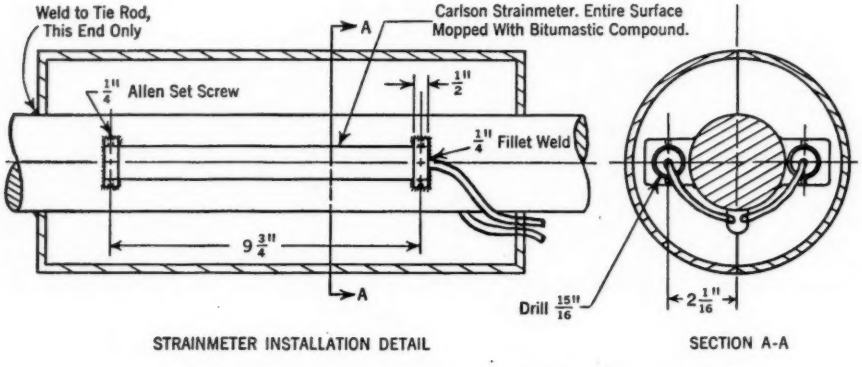


FIG. 7.—INSTALLATION DETAIL FOR CARLSON STRAINMETERS

of the two coils and their total resistance in series; change in resistance ratio is a linear function of strain, and change in total resistance is a linear function of change in temperature. The strainmeters, because of their successful record of strain measurement over long periods of time under field conditions, were adopted in preference to a bonded type of strain gage to assure longer life to the installation and to avoid uncertainties as to waterproofing and as to possible shifts in zero reading.

Fig. 7 shows the method of attaching the strainmeters to the tie rods—an arrangement designed to cancel out the effects of bending. Precision of individual readings corresponded to a stress of about 150 lb per sq in. Before taking initial readings, all tie rods in the vicinity of the test stations were adjusted as nearly as possible to the same length by the turnbuckles.

[3] "Five Years Improvement of the Elastic Wire Strainmeter," by R. W. Carlson, *Engineering News-Record*, Vol. 114, May 16, 1935, pp. 696–697.

[4] "Report on Instruments Used on Test Track at Hamilton Field," Report of Corps of Engrs., South Pacific Div., San Francisco Dist., December, 1946.

Difficulty was encountered with the strainmeters because of failure adequately to protect the lead wires against tensile forces resulting from the drag of the settling fill. When the fill level reached El. +8, the strainmeters began to become inoperative one by one until, when the fill had reached its full height there were only three strainmeters left—one at Station 27 + 30 and two at Station 30 + 30. Fortunately, the loss of strainmeters was a gradual process and it was possible to continue computations of tie-rod tensions by using data from one strainmeter on a rod. For these computations it was assumed that any strains resulting from bending in the horizontal plane were not significant except under the lower tensions.

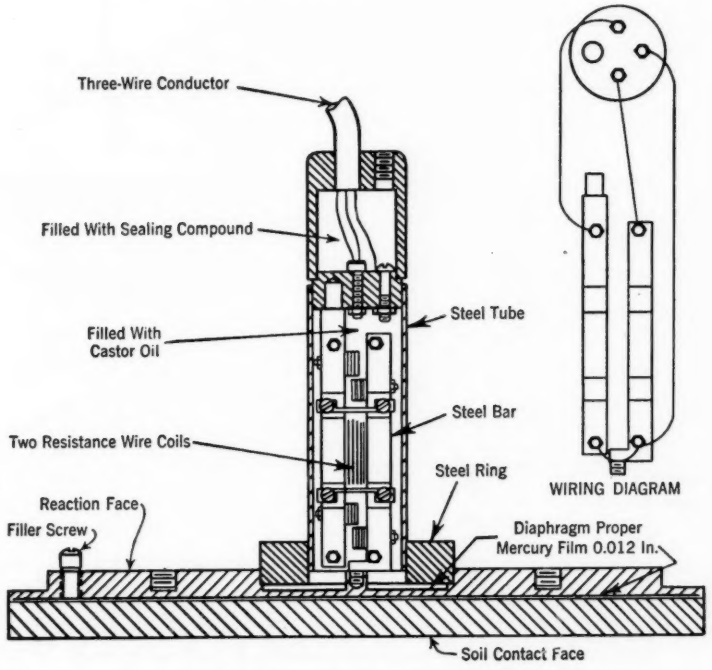


FIG. 8.—CARLSON STRESSMETER

Soil Pressure.—Nine stressmeters,[4,5] with ranges of 20 lb per sq in. and 40 lb per sq in. were used for the measurement of soil pressure against the bulkhead. These instruments, which have a successful record of stress measurement in concrete, and previously have been used with soil on several occasions, were specially adapted to this soil pressure problem. Pressure on the soil contact face (Fig. 8) causes pressure in the film of mercury which in turn deflects the "diaphragm proper." This deflection is measured with a strainmeter unit. The least reading corresponds to a pressure of less than 50 lb per sq ft.

[5] "Development and Analysis of a Device for Measuring Compressive Stress in Concrete," by Roy W. Carlson, thesis presented to the Massachusetts Institute of Technology, Cambridge, Mass., in 1939, in partial fulfilment of the requirements for the degree of Doctor of Science.

The stressmeters were calibrated for soil pressure in a testing machine using a stiff rubber pad[4] about 1 in. thick between the soil contact face and the testing machine weighing table. Load was uniformly distributed over the "reaction face" (Fig. 8). Calibration for water pressure was carried out at the test site by suspending the stressmeters in a cloth bag at measured distances below the water surface; fluid pressure then acted uniformly over the entire outside surface of the stressmeter. Four or five corresponding values of pressure and resistance ratio were determined in each case. Loading and unloading portions of the calibration cycles checked closely. Recalibrations checked to about 1% for soil pressure and to about 2% for water pressure. Fig. 9 gives calibration curves for a typical stressmeter.

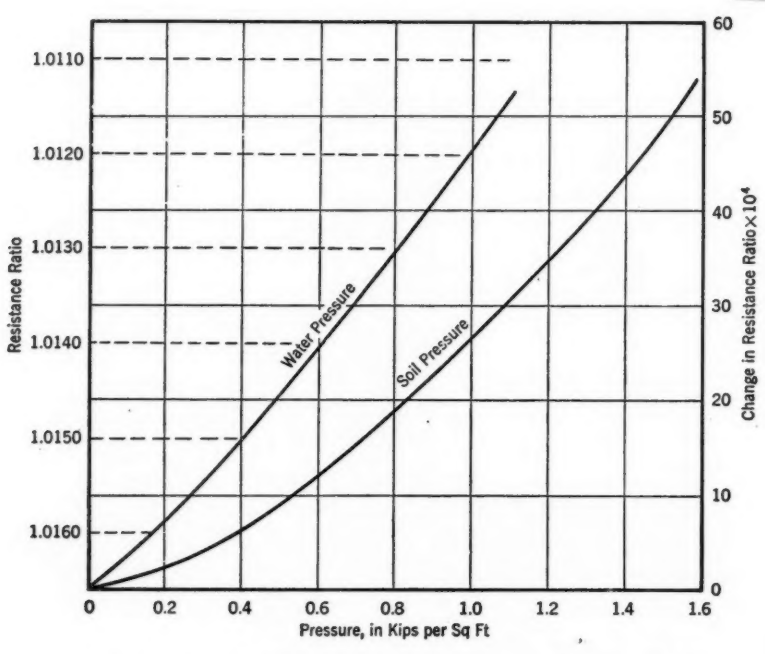


FIG. 9.—TYPICAL STRESSMETER CALIBRATION CURVES (STRESSMETER P-4)

The stressmeters were mounted in 22-in. steel plates as shown in Fig. 10. Reaction for the pressure was provided by two steel bars which also served to stiffen the plate; because of the considerable stiffness of the reaction face of the stressmeter, these bars were considered to provide the same support as used in the soil pressure calibration. The steel plates were held in the bulkhead troughs by spring friction clips. After the stressmeters had been installed, the steel plates (Fig. 6) were lowered into position, pressed into the bulkhead troughs by a diver, and securely welded to the bulkhead at their tops only.

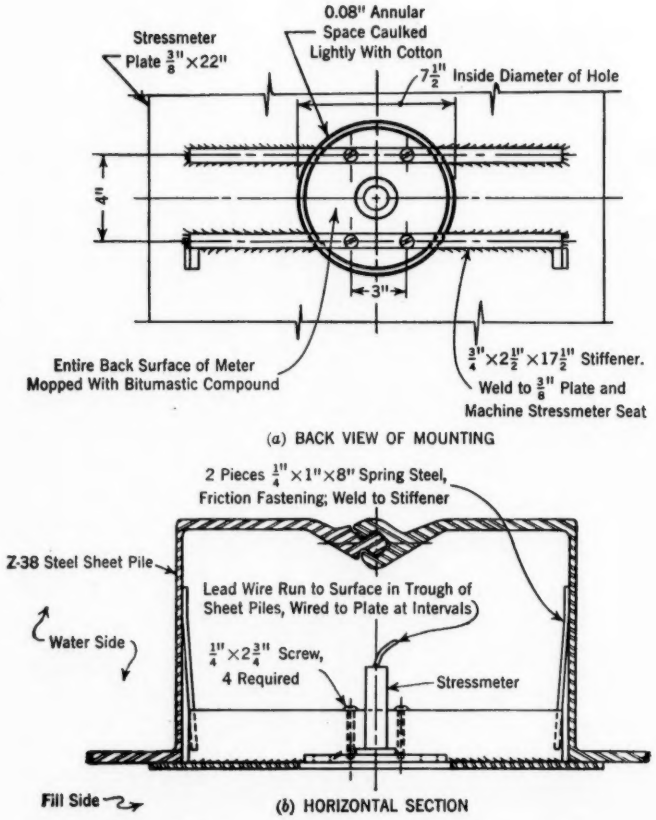


FIG. 10.—INSTALLATION DETAIL FOR CARLSON STRESSMETER

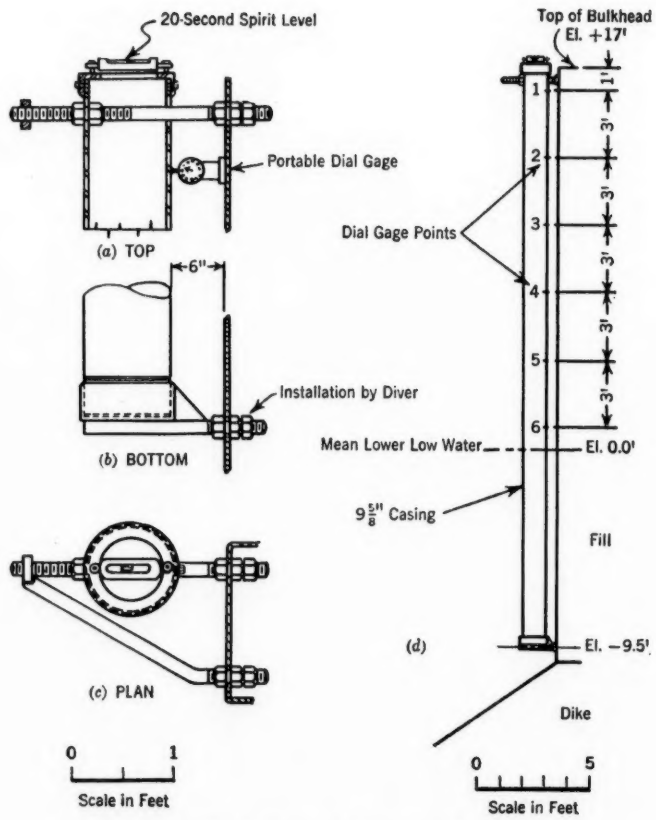


FIG. 11.—DEFLECTION MEASURING SYSTEM

A number of difficulties (most of which were overcome) were encountered in the installation and use of the stressmeters—early unsteadiness of zero readings, inadequacy of friction clips, damage to stressmeters during installation, and broken lead wires. During the placing of the dike, the lower inboard plate was dislodged to the position shown in Fig. 6, about 6 in. from the bulkhead face. The probable validity of measurements in the dike was questioned from the first because of the large size of dike fragments.

TABLE 2.—TIE-ROD TENSIONS, IN KIPS

Date (1949)	(a) STATION 27+30					(b) STATION 30+30					
	Tide Elevation[a]		Station[b] 27+18	Station[b] 27+42	Composite	Tide Elevation[a]		Station[b] 30+18	Station[b] 30+30	Station[b] 30+42	Composite
	Inside (1)	Outside (2)	(3)	(4)	(5)	Inside (1)	Outside (2)	(3)	(4)	(5)	(6)
April—											
15	....	....	6	10	8	....	....	−5	5	1	0
19	....	....	6	11	8	....	....	−7	2	2	0
19	....	....	4	11	8	....	....	−5	3	2	0
23	4.2	1.7	22	24	23	5.1	1.9	8	22	7	12
26	5.0	2.0	28	24	26	5.0	2.0	3	17	11	10
26	5.3	4.0	18	25	22	5.0	3.0	9	21	13	14
26	5.5	6.0	24	27	26	5.6	5.6	3	7	0	3
29	5.6	3.9	30	23	26	5.2	3.1	4	26	21	17
May—											
3	5.5	4.4	32	30	31	5.5	4.5	19	33	26	26
6	6.6	4.4	46	42	44	6.6	3.7	11	40	34	28
10	8.2	2.5	30	53	42	8.2	3.7	33	48	41	41
14	10.5	3.7	....	62	58	10.5	3.7	49	122	62	78
17	....	4.8	72	76	74	....	4.7	48	109	....	73
20	....	4.0	138	97	118	....	4.0	63	162	67	97
24	....	4.4	....	155	155	....	5.0	73	183	63	106
June—											
3	....	5.7	....	182	182	....	6.0	71	194	55	107
11	....	0.5	....	205	205	....	1.0	....	266	76	153
July—											
12	....	3.6	....	209	209	....	3.8	....	255	49	135
August—											
5	....	8.0	....	198	198	....	8.0	....	257	55	139
5	....	5.5	....	197	197	....	5.5	....	265	57	143
5	....	4.0	....	214	214	....	3.7	....	267	62	147
6	....	2.0	....	212	212	....	1.6	....	269	64	149
6	....	0.7	....	219	219	....	0.7	....	269	66	150
17	....	6.0	....	229	229	....	5.6	....	291	83	169
24	....	2.8	....	....	....	....	3.7	....	297	92	176
October—											
5	....	3.0	....	....	....	....	3.2	....	293	....	172

[a] Both inside and outside free water levels (in feet) are given where appropriate. [b] A horizontal rule in a column indicates where one of the two strainmeters went out of action. Increases from this point are based on one strainmeter only. Meters S-3 and S-4 at Station 27+30 gave erratic values that have been discarded.

Water Pressure.—The pressure of water against stressmeters and against the bulkhead was determined during the early stages of filling by direct observation of tide elevation. After the closure of the southeast corner of the pier the water rose inside, and both inside and outside water elevations were observed. After the fill rose above the inside water level, the tide data were supplemented at Station 27 + 30 with piezometer readings at three elevations (Fig. 6) on the back face of the bulkhead. Three additional piezometers were installed to provide information about ground water in the fill. The

piezometers consisted of pipes running to simple filter-type pore pressure points (Fig. 3).

Bulkhead Deflection.—Deflection of bulkhead points was determined by the vertical reference beam shown in Figs. 7 and 11. The beam was plumbed by a precise level tube mounted at the top. Relative deflection measurements

TABLE 3.—SUMMARY OF MEASUREMENTS, STATION 27 + 30

Date (1949)	Elevations,[a] in Feet			Stress[b] σ_v at El. −10	Tie-rod tension (kips)	Horizontal deflection at El. −9.5 (in.)	Soil Pressure in Kips per Square Foot						
		Tide					Inside Face					Outside Face	
	Fill	Inside	Outside				El. +12.3	El. +7.3	El. +0.7	El. −7.3	El. −14.9	El. −22.8	El. −32.8
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
February—													
17	−32	....	....	0	....	....	....	....	....	....	0.00	0.00	0.00
25	−30	....	....	0	....	....	....	....	....	....	0.00	0.56	0.38
March—													
1	−29	....	....	0	....	....	....	....	....	....	0.22	0.70	0.26
4	−27.5	....	....	0	....	....	....	....	....	....	0.40	0.69	0.27
8	−27	....	....	0	....	....	....	....	....	....	0.50	....	....
13	−26	....	....	0	....	....	....	....	....	....	0.73	0.86	0.36
17	−22	....	....	0	....	0.02	....	....	....	....	0.77	0.84	0.38
22	−22	....	....	0	....	0.05	....	....	....	....	1.40	0.94	0.50
26	−21	....	....	0	....	0.00	....	....	....	....	1.00	0.99	0.33
29	−21	....	....	0	....	0.00	....	....	....	....	0.96	1.15	0.30
April—													
2	−18	....	....	0	....	0.11	....	....	....	....	0.80	0.98	0.31
6	−17.5	....	....	0	....	0.02	....	....	....	....	0.80	1.03	0.28
9	−14	....	....	0	....	0.04	....	....	....	....	0.90	1.02	0.40
12	−11	....	1.2	0	....	0.09	....	....	....	....	1.20	1.32	0.53
15	− 8.5	....	2.7	0.08	8	0.42	....	....	....	....	1.10	1.22	0.42
19	− 7	....	3.5	0.17	8	0.52	....	....	....	0.00	1.06	1.20	0.45
23	− 1	4.2	1.7	0.50	23	0.82	....	....	....	0.17	1.33	1.47	0.60
26	3.2	5.3	4.0	0.73	25	1.89	....	....	0.00	0.55	1.19	1.61	1.05
29	5.8	5.6	3.9	0.88	26	2.15	....	....	0.11	0.50	1.06	1.38	1.27
May—													
3	7.5	5.5	4.4	1.00	31	2.48	....	0.00	0.00	0.76	1.06	2.40	1.12
6	7.8	6.6	4.4	1.07	44	2.57	....	0.08	0.00	0.55	1.00	1.47	0.87
10	9.4	8.2	2.5	1.17	42	2.78	....	0.08	0.26	0.86	0.98	1.90	1.04
14	12.0	10.5	3.7	1.42	58	....	0.00	0.38	0.56	1.04	1.62	2.30	1.12
17	12.1	....	4.8	1.52	74	3.39	....	0.73	0.68	1.25	1.84	1.98	0.98
20	13.1	....	4.0	1.59	118	3.33	....	0.26	0.85	1.48	1.59	1.60	0.96
24	16.0	....	4.4	1.94	155	3.42	....	0.55	0.23	1.14	1.70	1.58	0.95
June—													
3	16.3	....	5.7	2.01	182	3.36	....	0.67	1.30	1.04	1.74	1.35	0.60
11	16.5	....	0.5	2.14	205	3.40	....	0.70	1.18	0.71	2.52	1.88	0.85
July—													
12	16.5	....	3.6	2.07	209	3.46	....	1.28	0.00	0.58	2.53	1.58	....
August—													
5	16.5	....	8.0	1.96	198	3.37	....	1.79	0.00	0.68	2.79	0.80	0.6
6	16.5	....	0.7	2.09	219	3.37	....	1.85	0.00	0.24	2.33	0.95	....
17	20.5	....	6.0	2.43	229	3.37	2.70	2.36	0.00	0.55	1.62	0.69	1.07
24	20.5	....	2.8	2.49	....	3.41	2.85	2.60	0.00	0.48	1.30	0.87	....
October—													
5	20.5	....	3.0	2.48	....	3.63	....	....	0.00	0.57	1.85	0.74	....

[a] Both inside (Col. 3) and outside (Col. 4) free water levels are given where appropriate. [b] In Col. 5, σ_v is the computed vertical soil pressure, in kips (thousands of pounds) per square foot.

were made to the nearest 0.01 in. with a portable dial gage. Although provision was made for transit observations of anchor deflection using a special fin (Fig. 4) bolted to the cap and projecting up through the fill, these were insufficiently precise; thus in the calculations it has been necessary to neglect deflection of the anchor and wale.

Data Presentation

Data on Bulkhead Performance.—Results of measurements are presented in Tables 2 through 5 and in Fig. 12. Tie-rod tension measurements at both test stations are given in Table 2 and are summarized in Tables 3 and 4. Because of the successive breaking of strainmeter leads, the composite values in Table 2 become gradually more approximate with the passage of time, beginning about May 6.

TABLE 4.—Summary of Measurements, Station 30 + 30

Date (1949)	Elevations, in Feet			Stress[b] σ_v at El. −10	Tie-rod tension (kips)	Horizontal deflection at El. −9.5 (in.)
	Fill	Tide[a]				
		Inside	Outside			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
March—						
1	−35	....	....	0	....	....
17	−30.5	....	....	0	....	0.00
22	....	....	....	0	....	0.02
26	....	....	....	0	....	0.13
29	....	....	....	0	....	0.08
April—						
2	....	....	....	0	....	0.17
6	−28.5	....	....	0	....	0.13
9	−26	....	....	0	....	0.14
12	−25	....	....	0	....	0.10
15	−23	....	....	0	0	0.14
19	−22	....	....	0	0	0.16
23	−20	5.1	1.9	0	12	0.86
26	−14.5	5.0	3.0	0	10	1.16
29	− 6	5.2	3.1	0.22	17	1.60
May—						
3	− 3	5.5	4.5	0.38	26	2.02
6	0.5	6.6	3.7	0.58	28	2.28
10	4.5	8.2	3.7	0.80	41	2.58
14	9.0	10.5	3.7	1.05	78	....
17	10.2	....	4.7	1.38	73	3.57
20	12.0	....	4.0	1.62	97	3.66
24	12.7	....	5.0	1.63	106	3.69
June—						
3	12.5	....	6.0	1.56	107	3.86
11	15.5	....	1.0	2.13	153	....
July—						
12	15.5	....	3.8	1.99	135	....
August—						
5	16.0	....	8.0	1.83	139	3.69
6	16.0	....	0.7	2.20	150	3.69
17	21.5	....	5.6	2.53	169	3.67
24	21.5	....	3.7	2.62	176	3.67
October—						
5	21.5	....	3.2	2.65	172	3.92

[a] Both inside and outside free water levels are given where appropriate. [b] In Col. 5, σ_v is the computed vertical soil pressure, in kips per square foot.

Tables 3, 4, and 5 summarize all measurements at the two stations. Free body diagrams of the bulkhead at Station 27 + 30 on 19 dates through August 17 are drawn in Fig. 12. Tie-rod tension values have been divided by six for these diagrams to give anchor pull per linear foot of bulkhead. Measurements of water pressure by piezometers have been used in computing soil pressures from stressmeter readings. Hydrostatic excess, which was small in

general, and therefore has not been shown in Fig. 12, can be computed from data in Table 5.

Data on Settlement.—Unfortunately, direct measurements of fill settlement as a function of time and of position in fill were omitted. Sufficient indirect data are available, however, to permit approximate quantitative determinations. Measurement of movements of piezometer pipes showed settlements adjacent to the bulkhead ranging up to 0.6 ft for piezometer H-6; this amount apparently occurred below El. −9 where the horizontal pipe to H-6 left the bulkhead, Fig. 6. Cracks of considerable magnitude occurred in the fill over the anchor caps, as illustrated in Fig. 4, which indicate sizeable settlements.

TABLE 5.—PIEZOMETER READINGS, STATION 27 + 30
(FEET ABOVE MEAN LOWER LOW WATER)

Date (1949)	Time	Tide[a]	Piezometer						Inside water level	Fill elevation (ft)
			H-1	H-2	H-3	H-4	H-5	H-6		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
April—										
19	3:00 p.m.	4.2 (r)	....	4.0	4.4	4.1	4.0	4.0	Tide	−7
23	12:00 p.m.	1.5 (f)	....	3.3	2.8	2.2	4.8	3.8	3.4	−1
May—										
3	3:00 p.m.	4.4 (r)	6.5	4.3	4.2	4.5	4.5	4.5	5.5	7.5
6	3:30 p.m.	4.6 (r)	5.8	4.5	4.9	5.1	5.1	4.9	6.6	7.8
10	2:30 p.m.	2.7 (r)	7.1	4.4	2.3	4.4	5.7	5.5	8.2	9.4
14	3:15 p.m.	3.7 (f)	7.5	5.4	3.0	5.2	5.6	5.4	10.5	12.0
17	3:20 p.m.	4.8 (r)	5.8	5.8	4.5	5.7	6.3	5.7	....	12.1
20	2:10 p.m.	3.8 (r)	7.1	5.2	3.7	5.0	5.9	5.8	....	13.1
24	3:15 p.m.	4.2 (r)	5.6	4.6	3.7	4.9	5.9	5.6	....	16.0
June—										
3	3:30 p.m.	5.9 (r)	<5.4	5.0	5.6	6.3	6.3	5.1	....	16.3
11	5:30 a.m.	0.9 (r)	<5.4	2.6	1.0	2.3	2.6	4.6	....	16.5
July—										
12	4:30 p.m.	3.7 (r)	<5.4	4.1	3.5	3.5	3.8	3.8	....	16.5
August—										
5	7:10 p.m.	8.0 (f)	5.4	6.2	7.0	6.4	5.9	5.5	....	16.5
5	10:00 p.m.	4.8 (f)	5.5	6.0	5.1	5.3	5.6	5.6	....	16.5
5	10:40 p.m.	4.0 (f)	5.5	5.5	3.9	4.4	5.4	5.4	....	16.5
6	12:05 a.m.	2.0 (f)	<5.4	4.6	2.3	3.3	4.0	4.2	....	16.5
6	2:00 a.m.	0.7 (f)	<5.4	3.5	1.2	2.5	3.0	3.1	....	16.5
17	4:40 p.m.	6.0 (f)	<5.4	5.3	5.3	5.3	5.5	5.0	....	20.5
17	5:45 p.m.	5.2 (f)	<5.4	5.3	5.2	....	....	....	....	20.5
24	4:00 p.m.	3.4 (r)	<5.4	4.1	3.3	3.4	3.8	4.1	....	20.5
October—										
5	3:00 p.m.	3.0 (r)	<5.4	4.3	4.6	4.5	4.7	4.2	....	20.5

[a] (r) denotes rising tide and (*f*) denotes falling tide.

Breakage of strainmeter leads seems attributable only to such a cause. Usual experience of the Long Beach Harbor Department with similar fills obtained from the same Los Angeles River delta soils has indicated settlements of the order of from 2% to 3% over periods of from six months to eighteen months; this would correspond to about 1 ft in six months at the Pier C test stations at locations away from the bulkhead and anchor.

No doubt the greater part of the differential settlement of fill with respect to the bulkhead is seated in the lower fill strata (Table 1) of very fine sand and silty clay, and in the silty upper stratum (Fig. 5) of the foundation soil. Rough computation using the compressibility data of Table 1 yields a value of about 1 ft for the differential settlement due to these strata.

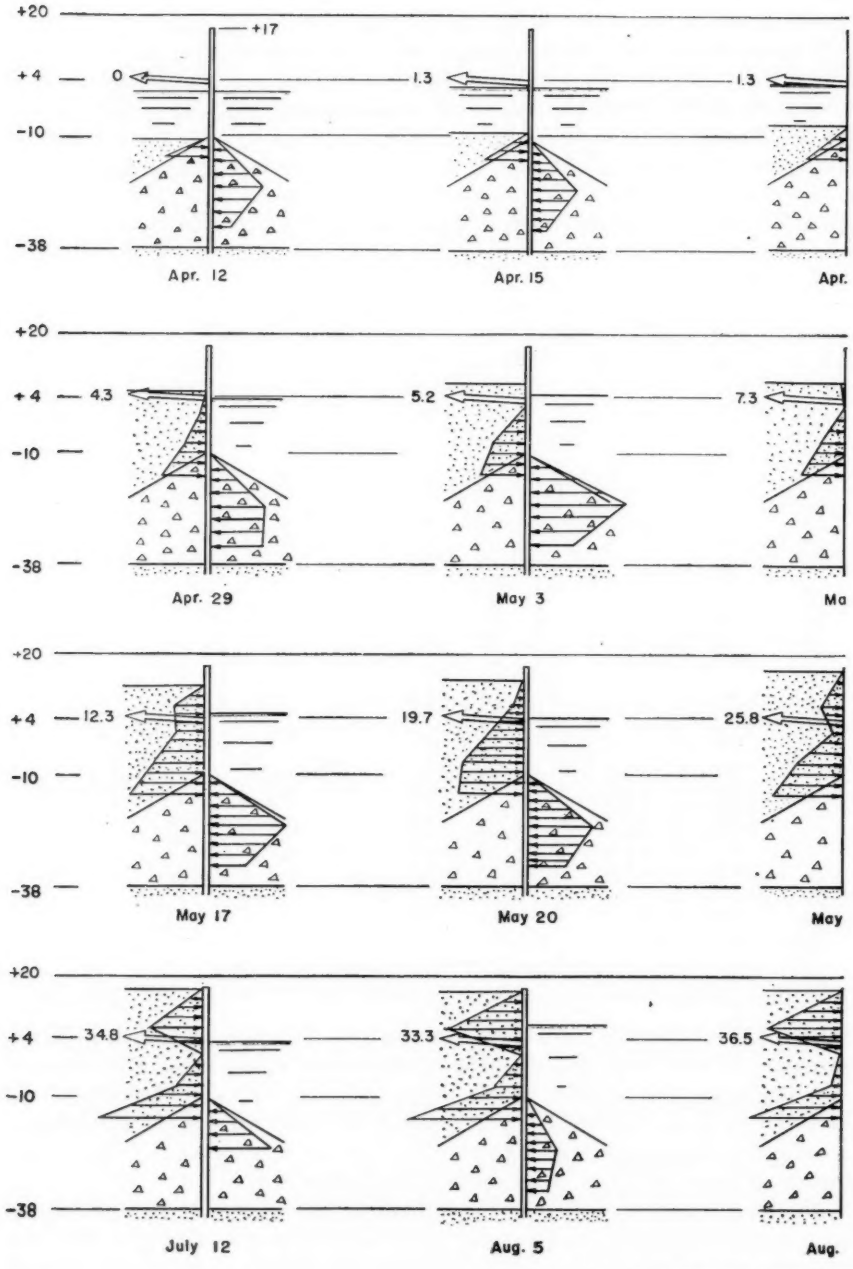
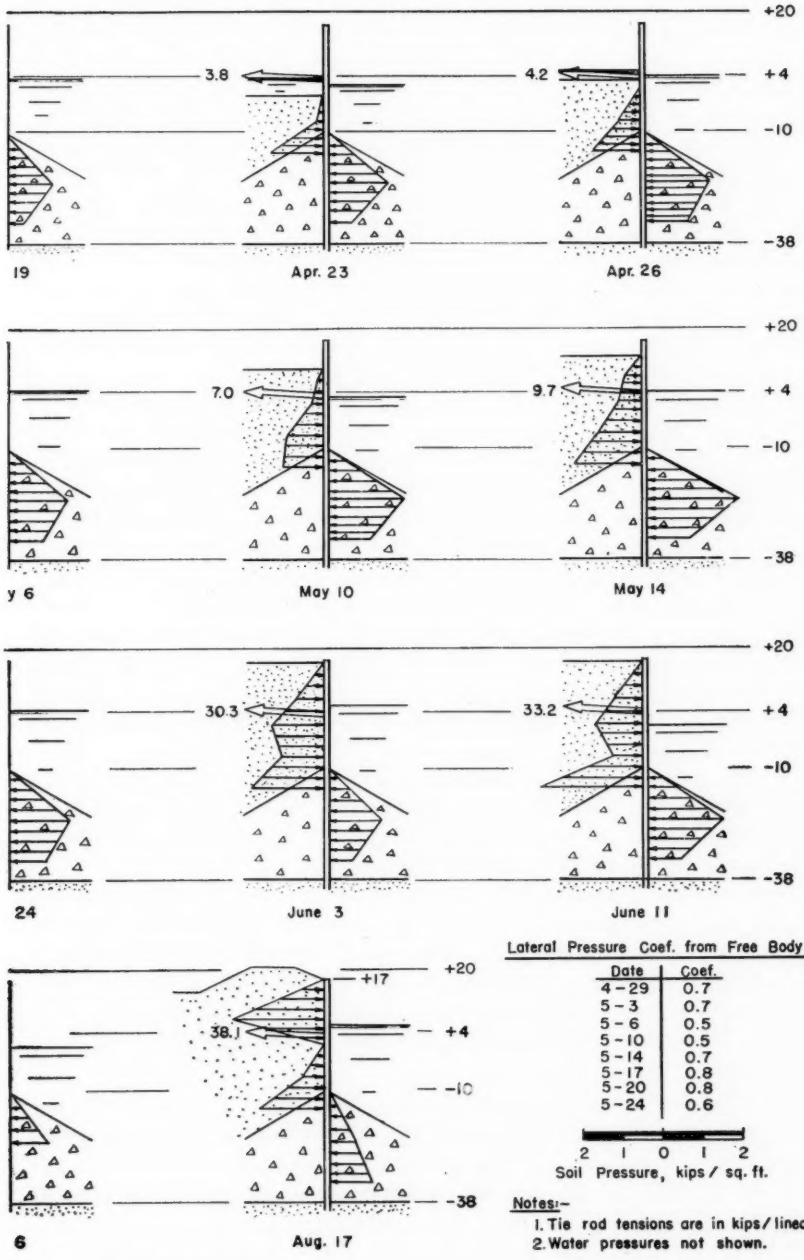


FIG. 12.—BULKHEAD FREE BODY



Lateral Pressure Coef. from Free Body Diagrams

Date	Coef.
4-29	0.7
5-3	0.7
5-6	0.5
5-10	0.5
5-14	0.7
5-17	0.8
5-20	0.8
5-24	0.6

2 1 0 1 2
Soil Pressure, kips / sq. ft.

Notes:-
1. Tie rod tensions are in kips / linear foot of wale.
2. Water pressures not shown.

DIAGRAMS, STATION 27+30

Bulkhead Performance During Filling

Outward Pressure of Fill.—In the early stage of filling, while the fill level is below the wale, a bulkhead in contact with soil deflects outward, away from the fill. This movement is suited to the development of a lateral pressure somewhere between the limits of active pressure and earth pressure at rest. "Earth pressure at rest," is the state of horizontal soil pressure existing in a horizontally infinite natural deposit. In the later stage of filling, observations and calculations show that the bulkhead above the wale deflects inward, toward the fill, tending to develop a lateral pressure between the limits of earth pressure at rest and passive pressure. Simultaneously the deflection below the wale continues outward. Should the bulkhead top be unprecedentedly high with respect to the wale, the directions of deflections above and below the wale would eventually reverse as the fill level rose. Presence of the dike does not influence this qualitative reasoning materially.

In general, then, one should look for pressures somewhat less than earth pressure at rest below the wale, and somewhat greater above. For small deflections, or with a rather fluid fill, the difference will not be easily detectable, and it is on this basis that a single value of the coefficient of lateral pressure has been sought from the Pier C bulkhead measurements. This coefficient is determined by two different methods from the stressmeter measurements and checked against deflection and tie-rod tension measurements.

Vertical soil pressures, computed from soil densities in Table 1, are given in Tables 3 and 4. This computation neglects friction between soil and bulkhead during the filling operation. Vertical pressure at El. −10 is used as the abscissa in several graphs. It serves as a convenient measure of the net soil weight above El. −10. Vertical pressure at any other level is equal to that at El. −10, plus or minus the net weight of soil between.

Fig. 13 shows the measured outward fill pressures at Station 27 + 30 plotted against vertical pressure computed for El. −10. The slopes of these "curves" are the lateral pressure coefficients, values of which are indicated on the figure, for the respective stressmeter locations.

The lateral pressure coefficient is obtainable in another way from Fig. 12. During the filling operation (to about May 24), these outward pressure distributions have shapes determined by the value of the coefficient. By the use of a template ruled with theoretical lateral pressure lines (broken at ground-water level) corresponding to various assumed lateral pressure coefficients, and assuming the soil submerged below El. +4, the values indicated in Fig. 12 were obtained.

As a result of the foregoing determinations, the lateral pressure coefficient of the fill material during the filling operation will tentatively be taken at 0.7.

Lateral Pressures in Dike.—Placement of the dike on both sides of the bulkhead was planned in an effort to reduce bending moments in the sheet piling. The outer portion was expected in effect to reduce the structural span of the piling. The inner part[6] was to absorb some of the lateral pressure due

[6] "Reduction of Lateral Cohesive Soil Pressure on Quaywalls by Use of Sand Dikes," by Harris Epstein, *Proceedings*, 2d International Conference on Soil Mechanics and Foundation Eng., Rotterdam, Holland, 1948, Vol. III, pp. 291–296.

to the rather fluid fill. The inner dike could not have been placed more strategically to perform this function, for the deposition of sediments (Table 1) placed the finer and more plastic strata at the bottom.

The dike exerted a theoretical initial lateral pressure on both sides of the wall about equal to the active pressure since it was placed at its natural angle of repose. Theoretically, again, as the fill rose in the pier, the outer dike lateral pressures increased in accordance with increasing deformation. The inner dike lateral pressures increased, corresponding with rising fill, to greater active pressures at the base and, in the upper part, to values probably in excess of active dike-material pressure. This is an extension of findings[7,8] and theory,[6]

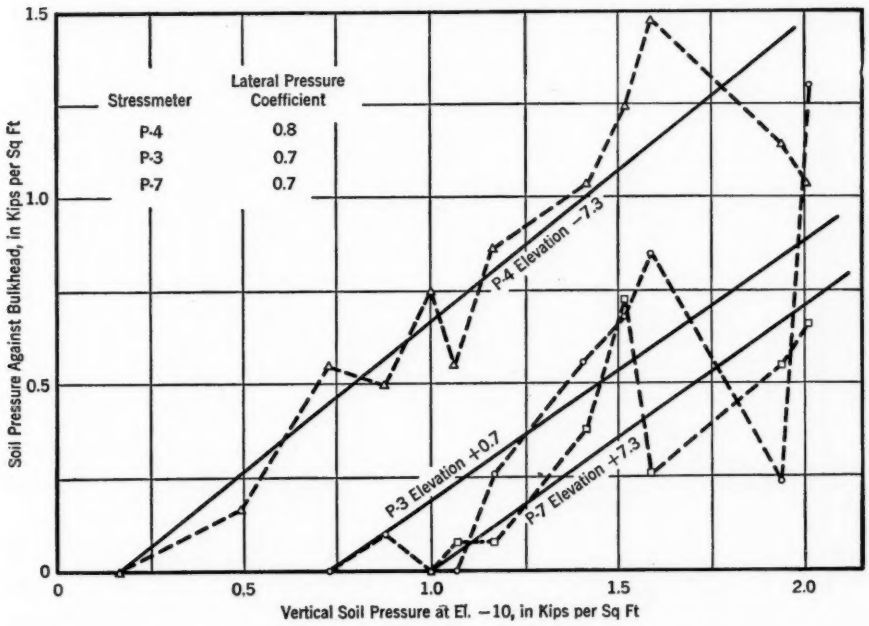


FIG. 13.—LATERAL PRESSURE COEFFICIENTS, DURING FILLING, FROM INDIVIDUAL STRESSMETERS

on the subject of dikes in fluid fills, necessitated for Pier C by the gradually increasing thickness of dike with depth combined with large depth of fill.

Because dike stressmeter data (Tables 3 and 4) were not as satisfactory as for the fill (probably due to the large size of fragments), full reliance cannot be placed on these observations in arriving at a picture of dike behavior; the foregoing theory must also be drawn upon. The initial measurements of dike pressure in the bulkhead are inconsistent with active pressures and are believed to be in error. However, subsequent small increases in stressmeter indications may possibly be rough approximations to the actual increases. Based partly on the theory and partly on rational interpretation of stressmeter data, the

[7] "Soil Mechanics, Foundations, and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, pp. 293–303.
[8] *Ibid.*, pp. 510–514.

following idealizations are made tentatively to facilitate checking the various types of measurements against one another:

1. For the inner dike, the lateral pressure at the base was taken as active, with a coefficient of 0.3; the lateral pressure at the top was taken as that due to fill, with a coefficient of 0.7; and a straight-line variation of pressure was assumed between.

2. For the outer dike, the lateral pressure was taken as active, with a coefficient of 0.3, until the fill reached El. +4 (see Table 3); for the fill above El. +4, the lateral pressure was taken as active, plus a uniform 0.5 kips per sq ft.

These idealizations are given in Fig. 14 for two levels of fill to supplement the pressure data obtained for the fill itself. Symbols for soil pressures, in kips

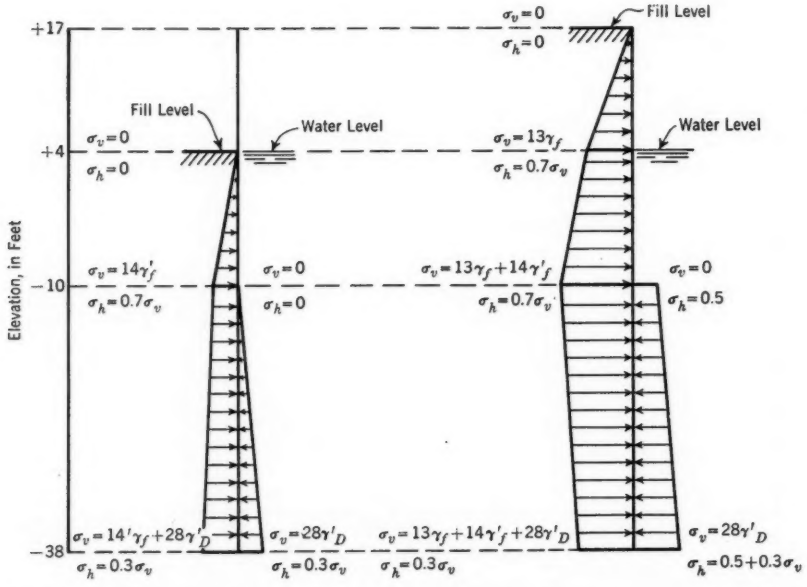


FIG. 14.—SOIL PRESSURES USED FOR COMPUTATION OF BULKHEAD DEFLECTION AND ANCHOR PULL

per square foot, and unit weights, in kips per cubic foot, are as follows: σ_v and σ_h denote vertical and horizontal soil pressure, respectively; γ_f and γ'_f denote the air-dry and submerged unit weights, respectively, of the fill material; and γ'_d denotes the submerged unit weight of the dike material. The unit weights were obtained from Table 1.

Bulkhead Deflection and Anchor Pull: Data Comparison.—Further data on lateral pressure are implicit in the bulkhead deflection and anchor pull measurements, which are of special importance because results are available from both of the test stations. These are now to be compared with the direct pressure measurements at Station 27 + 30, which are summarized in Fig. 14 for two levels of fill.

The method of computing deflections and tie-rod tensions from soil pressure measurements is shown in Fig. 15. It is idealized that the bulkhead is fixed against rotation at El. −38; confinement due to the dike and rigidity of the natural ground tend to provide this restraint. The statically indeterminate free body thus obtained has been analyzed according to the conventional theory of flexure, and curves of shear, moment, and deflection are given.

The first comparison is given in Fig. 16, where representative observed and computed profiles of the deflected bulkhead are superposed. The shapes of the curves, in so far as they correspond, are in fair agreement, but computed values are less than those directly observed.

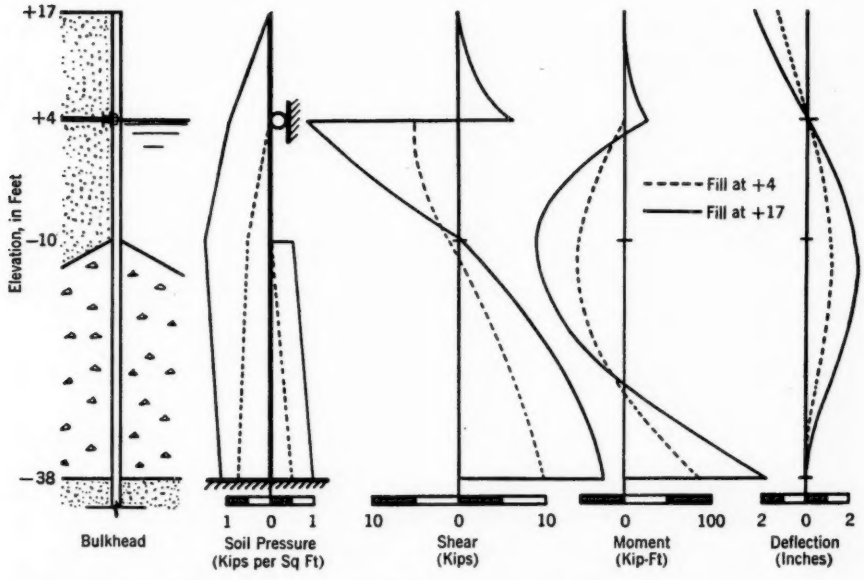


FIG. 15.—BULKHEAD DEFLECTION AND ANCHOR PULL COMPUTED FROM MEASURED SOIL PRESSURES

The most significant comparison is shown in Fig. 17, in which the results of Fig. 15 have been amplified slightly to give theoretical plots against vertical soil pressure at El. −10. Reasonable agreement between computed and measured tie-rod tensions is indicated for the period here considered—that is, during the filling operation. The large tensions at Station 30 + 30 in the early stages are attributed to the high inside water level during that period (Table 4). This influence is also felt in the observed deflections at Station 30 + 30. The hinge at the bottom of the deflection reference beam (El. −9.5) deflects about 25% less by computation than in fact, as shown by the upper curves of Fig. 17. Shapes of both computed curves compare reasonably well with the observed.

It is considered that these comparisons of data obtained during the filling operation are acceptably good. The check between tie-rod tensions and soil pressures (Fig. 17) is further evidence of the correctness of the previously

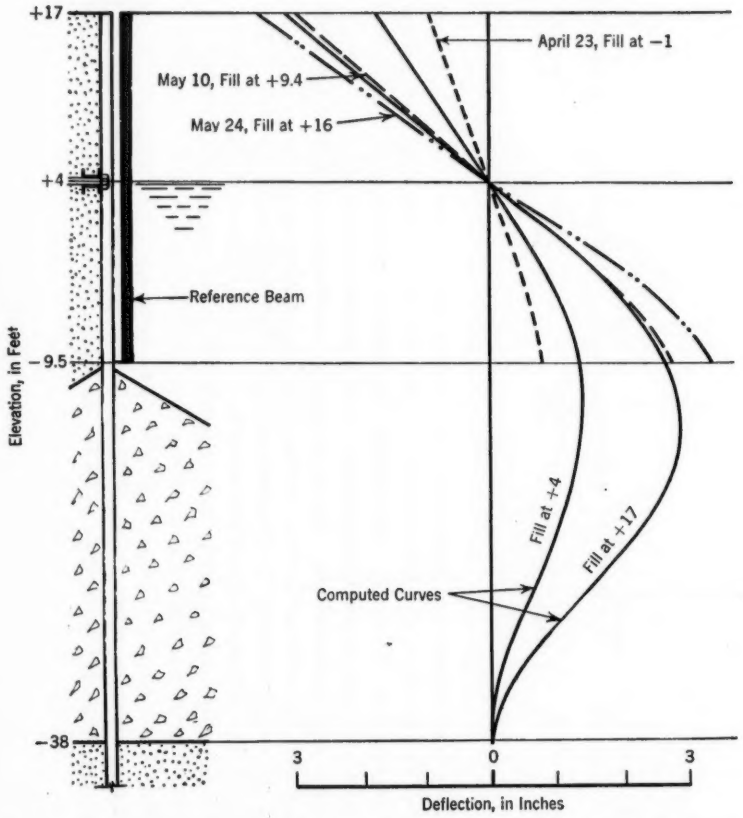


FIG. 16.—MEASURED DEFLECTION PROFILES VERSUS THOSE COMPUTED FROM MEASURED SOIL PRESSURES, STATION 27 + 30

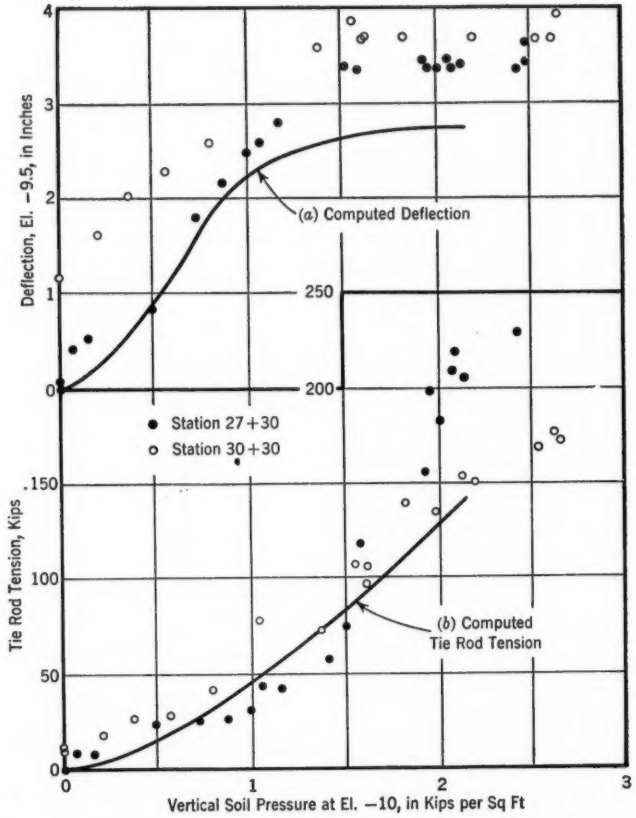


FIG. 17.—COMPARISON OF MEASURED DEFLECTIONS AND ANCHOR PULLS WITH VALUES COMPUTED FROM MEASURED SOIL PRESSURES

deduced lateral pressure coefficient of 0.7 and to a degree validates the assumptions relating to the dike and to the condition of base support. The 25% discrepancy of the deflections computed from soil pressures (Figs. 16 and 17) relative to those observed directly may quite possibly be due to a small outward deflection of the bottom of the bulkhead.

Other Pressure Distributions

Nonproportional Pressure Distribution After Filling.—Beginning on about June 11, the lateral pressure distribution curve at Station 27 + 30 began to assume a significantly different shape (Fig. 12). Lateral pressure was no longer directly proportional to the weight of the overlying fill. Pressures between the wale and the top of the dike dropped sharply, whereas those above this region increased. Simultaneously, the tie-rod tension at Station 27 + 30 increased considerably. However, the tie-rod tension at Station 30 + 30 did not increase correspondingly (Fig. 17). Deflections remained constant after the completion of filling.

Explanation of the pressure redistribution at Station 27 + 30 is believed to lie primarily in the settlement of the fill below the tie rods. As estimated above, the compression of the strata below the tie-rod level, at locations away from the bulkhead and anchor, was about 1 ft during the period of the tests. Settlement adjacent to the bulkhead was only about half this distance which was due partly to the dike, but largely to the supporting capacity of the tieback system. There is a relative absence of opportunity for lateral deflection at the tie-rod level, so that the rods were caused to act as "cables" supporting a transverse load, with a resultant increase in stress.

The deflection of the bulkhead is discounted as a cause of this pressure redistribution, as essentially no change in deflection was observed after the completion of filling. Curve (*a*), Fig. 17, shows this condition clearly.

This result is new in so far as quantitative observation is concerned although experience has led at least one progressive engineer[9] to provide special features in his bulkhead design to prevent the loading of tie rods.

A tentative design consideration of importance is that maximum bending moment and tie-rod tension probably do not occur simultaneously in any filled anchored bulkhead. The former occurs between the wale and the ground during filling, and results from the high lateral pressure coefficient which is approximately uniform before settlement of the upper fill. The latter occurs after the settlement of the fill due to the supporting, on the tieback system, of a part of the weight of fill above the tie rods. Conceivably, with the passage of years, the pressure distribution might trend toward a proportional one once more, although with a smaller lateral pressure coefficient.

The seeming absence of this interesting finding at Station 30 + 30 could be related to the high inside water level during the early stages of filling at that station.

Surcharge.—The effect of a 4-ft surcharge close to the bulkhead at Station 27 + 30 on pressure distribution is seen by comparison of the August 5 and

[9] "Soil Mechanics, Foundations, and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, p. 514.

August 17 free body diagrams (Fig. 12). Increase in pressure above the wale, and in the tie-rod tension, indicates that this surcharge has been carried essentially by the wale-and-tie-rod system. Increase in tie-rod tension due to this action of the tie rods as "cables" is directly related to the increase in lateral pressure. This finding is checked qualitatively by the measurements at Station 30 + 30, where tie-rod tension increased slightly more (Tables 3 and 4) due to a 5.5-ft surcharge. This is significant for design purposes as it indicates that surcharge should be especially investigated for its effect on tie-rod tension.

Tide Fluctuation.—Table 5 yields some results regarding water pressure fluctuations in the dike and the fill. Noting the close comparison of readings of piezometer H-3 (which was in the dike) with the tide levels, it is seen that the combination of dike and sheet piles was very pervious. The dike pore pressures were nearly equal to the free outside water pressure at all times, even when a very high inside free water level prevailed, as on May 10 and 14.

The August 5 and 6 readings (Table 5) are most descriptive of the effect of tide variation on pore pressures. Piezometer readings in the fill were roughly the same regardless of location, and ranged from about +6 to about +3 while the tide dropped from +8.0 to +0.7.

Changes in bulkhead loading due to this same large range of tide are given for a typical case in Fig. 12, August 5 and 6, at Station 27 + 30. Hydrostatic excess, which acts inward on August 5 and outward on August 6, is not shown in Fig. 12 but may be obtained from data in Table 5. Tie-rod tension increases about 15 kips (averaging Stations 27 + 30 and 30 + 30) with falling tide, while the changes in hydrostatic excess roughly balance those in soil pressure. The indicated changes, although appreciable, are small in comparison with total pressures and tie-rod tensions. For construction of the type at Pier C it would be overconservative to design in the usual fashion for the low tide condition, assuming water behind the bulkhead to be at high tide level.

Summary of Test Results

During the filling operation the pressure of fill against the bulkhead was approximately in proportion to the weight of the overlying fill. The proportionality constant, or coefficient of lateral pressure, was found to be about 0.7 for this predominately fine sand material. Internal pressure distribution not in proportion to the weight of overlying fill developed after the completion of filling. Pressures above the wale increased sharply while those below decreased. This condition is attributed mainly to the approximately 1-ft settlement of the fill and accompanying partial support of the upper part of the fill on wale and tie rods. This result was obtained at the primary test station only.

Maximum deflections were approximately 3.6 in. outward at the top of the dike and 3.9 in. inward at the top of the bulkhead, assuming no deflection of the wale. No reversal in direction of the deflections occurred at any time. Deflection changes ceased upon completion of filling. Observed deflections at the two test stations agreed reasonably closely. Predictions of deflections, based on soil pressure measurements and a structural analysis of the bulkhead, agreed qualitatively but were about 25% low.

Maximum tie-rod tension was about 200 kips. This occurred near the end of the tests, under maximum influence of pressure redistribution. Observed tie-rod tensions agreed reasonably closely at the two test stations. They agreed well with values predicted from measured soil pressures and bulkhead structural analysis.

The part of the dike inside the bulkhead was effective in reducing outward pressures. Its vertical position is such as to be of maximum efficiency, since the finer fill strata are in the lower depths. The part of the dike outside the bulkhead provided some passive resistance and served to confine the bulkhead foundation.

The surcharge piled close to the bulkhead after completion of filling was carried principally by the tieback system, pressures below the wale being not much affected.

Pore pressures in both inner and outer dikes were nearly equal to the free outside water pressures at all times. The extreme range of pore pressure fluctuation in the fill was about 3 ft of water and was roughly the same at all locations. Tie-rod tension increased about 15 kips in one case due to a 7-ft drop of tide.

Suggestions for Future Bulkhead Field Studies

Experience with the conduct and interpretation of the Pier C bulkhead tests has led to nine suggestions to future field investigators of anchored bulkheads, as follows:

1. An adequate system of survey control should be established, with better provision for observing total horizontal and vertical deflections.
2. Fill settlement should be observed with permanent reference points established both in and on the various soil strata.
3. Borings should be made in the fill and at intervals during fill deposition and "undisturbed" samples taken to determine density and shear strength.
4. Test stations, both with and without a dike, should be included.
5. Special attention should be given to tie-rod tension measurement, as this simple and inexpensive determination is probably the most significant of the various indicators of anchored bulkhead performance. Lead wires should be run in pipe, or otherwise protected from the settling fill.
6. Bulkhead bending strains could be remotely measured by a technique similar to that used for tie-rod tension.
7. The sensitivity and reliability of soil pressure measurements could be improved by a new design of the stressmeter mounting. A large plate in contact with the soil or the dike could be used to transmit force to one or more stressmeters. A simple inexpensive soil pressure cell would be a boon to the progress of soil mechanics.
8. Special attention to insuring long life to the instrumentation would make possible the study of the permanence of observed pressure distributions and of such influences as static and moving surcharge, impact, and seismic loadings. The changes accompanying passage of time, up to several years, are little known. The electric instruments used in these tests are suited to such long-time studies.

9. Creative thought is needed on the problem of the difficult and probably expensive quantitative observation of passive resistance phenomena at the base of a bulkhead.

Acknowledgment

The Long Beach Harbor Department designed the quay wall and purchased and installed the instruments. R. R. Shoemaker, M. ASCE, chief harbor engineer, jointly with the writer, conceived the test program. B. N. Hoffmaster, A. M. ASCE, and T. J. Thorley supervised the installation of instruments and contributed to the planning and execution. Exploration and testing of soils was carried out by L. T. Evans, A. M. ASCE, consultant to the harbor department.

The Department of Engineering, University of California, in Los Angeles, planned the instrumentation and conducted the tests and analyses. D. P. Krynine, M. ASCE, visiting professor, advised in the planning. The writer was in immediate charge. Students in engineering who participated were D. L. Wheeler, E. R. Hallett, V. S. Peterson, R. J. Reich, and E. Carlson, J. M. ASCE. The methods of attaching the strainmeters to the tie rods and of mounting the stressmeters were developed in consultation with Roy W. Carlson, M. ASCE, the inventor, and with David Pirtz, J. M. ASCE.

The Soil Mechanics Committee, Los Angeles Section, ASCE, gave considerable study to the planning of the test program and made numerous valuable suggestions. Committee members were T. R. Dames, M. ASCE (Chairman); F. J. Converse, M. ASCE; Mr. Evans; S. S. Green, M. ASCE; T. M. Leps, A. M. ASCE; D. F. Warren, J. M. ASCE; and the writer.

Interpretation of test results has benefited materially as a result of discussions with Mr. Krynine and with G. P. Tschebotarioff, M. ASCE.

To all these engineers, and to the many who helped in other ways, the writer extends his sincere thanks.